RESILIENT INFRASTRUCTURE
June 1–4, 2016

# FRP SHEAR CONNECTORS FOR REINFORCED CONCRETE COMPOSITE ELEMENTS

Jehad Alkatan,
University of Windsor, Canada

Amr El Ragaby
University of Windsor, Canada

Faouzi Ghrib
University of Windsor, Canada

## ABSTRACT

Currently, friction shear dowels (steel stirrups) are being used in bridge construction at the interface plane between the cast-in-place deck slab and the precast girder to ensure the composite action for stronger cross section. As the bridge deck slab deteriorated over time due to the environmental conditions, these shear steel dowels are also susceptible to sever corrosion and will need replacement, especially when de-icing salt is used. This results in gradual loss of the composite behavior and strength of the composite section. Fiber reinforced polymers (FRPs) reinforcements have shown to be an effective alternative to black steel as flexure and shear reinforcement for RC elements over the past 10 years. This research project aims to investigate a new application for FRP reinforcement as friction shear reinforcement between the precast and cast-in-place concrete members to overcome the corrosion problem and the corresponding high maintenance cost. This paper investigates the visibility of using FRP as shear-friction reinforcement (connectors) through push-off experimental tests on concrete specimens with Glass FRP (GFRP) shear connectors. The tested parameters included the shear connector's reinforcement ratio, geometry and layout. Test results are presented in terms of comparisons of the ultimate capacity and failure mode against steel reinforced ones as well as load-slip and load-strain relationships. Test results indicates an outstanding capacity and behavior of GFRP shear connectors compared to steel ones.

Keywords: Friction Shear, Composite Section, GFRP, Dowels

## 1. INTRODICTON

Composite construction is being extensively used in bridge design and construction for quite some time. In the earlier applications, composite beam referred to a concrete slab provided on top of a steel beam along with shear keys at the junction of the slab and the beam. However, composite concrete beams, which are a combination of cast-in-place slabs and precast girders, are widely used nowadays (Loov & Patnaik, 1994). Precast girders are usually fabricated in the industrial facilities before they are moved to their final position. The slab is then cast on top of the precast girder with its form supported by a fixed base in the case of shored construction or supported by the precast girder in unshored construction. When the composite action of the cast-in-place and the precast parts are ensured, the overall strength and stiffness of the composite section can be used. Therefore, lighter and shallower beams can be used leading to efficient and economical construction method. However, the reinforcement crossing interfaces remain the key parameter that allows the process of shear transfer along the joints to occur. Up to date, steel reinforcement crossing the shear plane between prefabricated girders and their cast-in-place flanges is being used according to different design models and expressions. Deterioration of the deck slab caused by the environmental and loading conditions results in extensive corrosion of the steel reinforcement between the slab and the girder especially when de-icing salt is used. This results in gradual loss of the monolithic behaviour and strength of the composite concrete beams. Epoxy coated steel reinforcement was proposed as a substitute of black steel at the joints, but it was shown to be ineffective in providing the desired corrosion resistance or in reducing the long-term maintenance cost (Pianca, Schell, & Cautillo, 2005). Fiber Reinforced Polymers (FRP) reinforcements, bars and stirrup, have shown to be an effective alternative to conventional steel as a flexural and shear reinforcement especially Glass FRP. In addition to their non-corrodible nature, the superior high tensile and bond strength as well as ease of handling of FRP reinforcement due to lightweight promoted their application in reinforced concrete structures. This research project aims to extend the application of FRP to be used as shear transfer reinforcement across interfaces of concrete cast at different times (Cold-joint condition). To explore the feasibility of the proposed technique and to optimize the effect of different design parameters, the entire research program was divided into two phases. In Phase-I, a series of push-off tests were conducted on large-scale concrete specimens with FRP reinforcement provided across the cold-joint interfaces of the push-off specimens. Phase II will further investigate the application through full-scale testing on composite concrete girders. This paper describes the details of the experimental program and the findings of Phase I.

## 2. PREVIOUS RESEARCH

Because of their sensitivity in the design of the composite structures, concrete-to-concrete connections have been justifiably and continuously studied over time. In the earlier practices of composite construction, it was believed that the shear transfer capacity of the concrete connections interfaces was equal to the shear strength of unreinforced beams (ACI Committee report 711, 1953). Accordingly, the interface was assumed to provide adequate shear strength if it was properly roughened and combined with shear keys. However, an extensive research was triggered by the appearance of the shear friction hypothesis. This hypothesis was first introduced by Birkeland and Birkeland (1966) and was later investigated and refined extensively by different researchers for different situations of concrete-to-concrete interfaces, over the past five decades. Push-off specimens were first introduced by Anderson (1960), since then, they were used extensively in the investigation of the shear transfer across concrete joints. A large number of design expressions were proposed for the ultimate shear transfer stress. However, the shear friction concept was the background on which the majority of these expressions were based on. There were six distinct studies that provided a significant advancement regarding this topic at the time of their disclosure, namely: (1) the first publication of the shear transfer theory by Birkeland and Birkeland (1966); (2) the modified shear friction theory by Mattock and Hawkins (1972), where the contribution of the concrete cohesion at the interface was incorporated; (3) the first non-linear equation for the shear transfer strength proposed by Birkeland (1968); (4) Loov (1968) was the first researcher to include the effect of the concrete strength in the evaluation of the shear strength of concrete interfaces; (5) the design expression developed by Walraven et al. (1987) after an extensive statistical analysis of a large number of push-off test results.; and (6) the first expression that accounted for the three shear transfer sub-mechanism (cohesion, friction and dowel action) introduced by Randi (1997). The main parameters that were shown to influence the shear transfer strength of concrete-to-concrete interfaces in the existing literature concerning this topic are: (a) reinforcement crossing the interface shear plane; (b) interface roughness; (c) normal stresses at the shear plane; and (c) the compressive strength of the concrete. However, no previous research or applications were found to utilize GFRP reinforcement as a shear transfer reinforcement in connections with concrete cast at different times.

## 3. EXPERIMENTAL PROGRAM

### 3.1 Test Specimens

The specimens reported in this paper were of large scale push-off type as shown in Figure 1, with a shear plane of 0.115 $m^2$. The specimens were loaded as indicated by the arrows shown in Figure 1, so the shear with out moment is produced in the shear plane. The investigated variables between the test specimens were, the GFRP reinforcement strength, ratio and geometry. Accordingly, the specimens were designated so the first letter in the specimen ID refers to the reinforcement type: S = steel and F = FRP, the second letter stands for the reinforcement geometry as follows: A =angle; H = headed bar; and S = stirrups, and the third character represents the number of the reinforcement parts of the same type and shape. All specimens were built using a custom from work to allow the formation of the cold-joint condition at their interfaces. Four specimens were cast at the time in the horizontal position as indicated by Figure 2. One part of each specimen was cast first with their interfaces left as-cast with no additional treatment. After three days, the interface was cleaned before the second part of each specimen was cast (see Figure 2). Additional longitudinal and shear steel reinforcement were provided away from the shear plane to avoid failures other than that along the shear plane. To allow for comparison, a control specimen was tested using one steel stirrup across its shear plane.

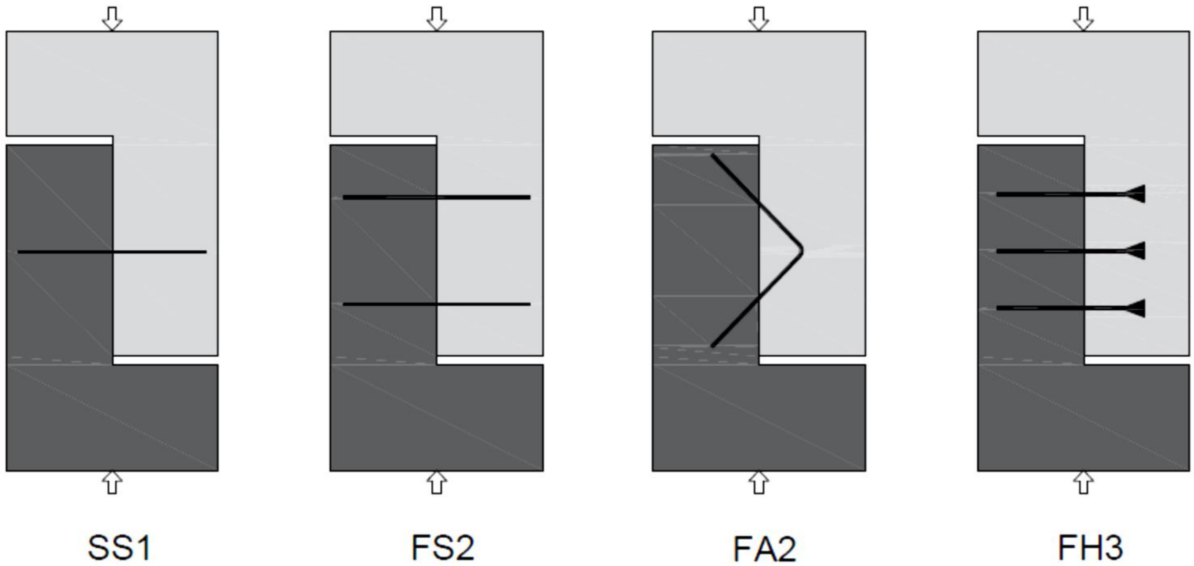


Figure 1: Push-off specimens

Figure 2: Test specimens casting

### 3.2 Material Properties

The specimens were fabricated using normal weight concrete with an average compressive strength of 50 MPa, at the day of testing. Since the concrete was cast at different times, a mechanical mixer at the laboratory of the University of Windsor was used to obtain consistent concrete properties. At least six 100x200 mm and three 150x300 mm cylinders were prepared for each concrete batch and cured under same conditions with their reference specimens to evaluate the compressive and tensile strength of the concrete in the testing day. The concrete compressive $f_c'$ and tensile $f_r$ strengths were evaluated in accordance to ASTM C39 (2015) and ASTM C496 (2011), respectively.

Figure 3 shows the shear transfer reinforcement used across the joints in this study. No.4-12M (V-ROD™) GFRP bars were used as a shear transfer reinforcement. All of the GFRP reinforcement had a sand-coated surface and was made of continues longitudinal fibers. GFRP stirrups and angles had a tensile modulus of approximately 50 GPa and the headed bars had a modulus of 60 GPa. Table 1 lists the mechanical properties of the GFRP reinforcing bars as per the supplier (V-ROD™ Canada).

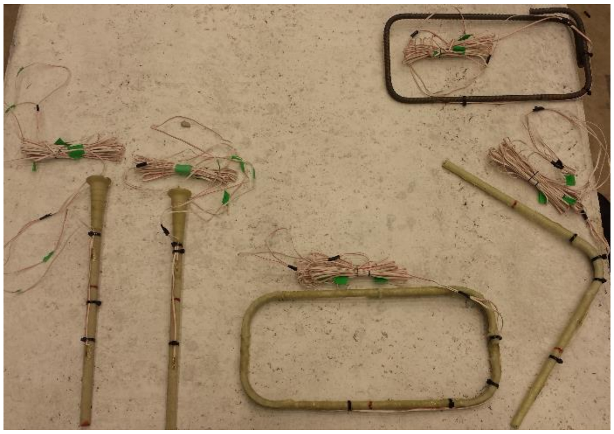

Figure 3: Shear transfer reinforcement

Table 1: Properties of GFRP bars

| GFRP Reinforcement | Nominal Cross-sectional area, mm2 | Tensile strength, $f_{Fu}$ (MPa) | Tensile modulus, $E$ (GPa) | Tensile strain (%) | Poisson's ratio |
|---|---|---|---|---|---|
| stirrup | 126.7 | 1140 | 50 | 2.17 | 0.26 |
| angle | 126.7 | 1140 | 50 | 2.17 | 0.26 |
| headed bar | 126.7 | 1312 | 60 | 2 | 0.26 |

### 3.3 Test Setup and Instrumentations

After the desired strength of 50 MPa was attained, each specimen was centered in its vertical position under the hydraulic jack of the testing machine as shown in Figure 4. Specimens were subjected to a monotonic loading up to failure at an average rate of 10 kN/min. In all cases, the relative slip and longitudinal strain of the shear transfer reinforcement at the level of the interface were measured. To do so, on either side of each specimen, a linear variable displacement transducer (LVDT) was provided, with its first end attached to one part of the specimen and the other end resting on an aluminum bracket fixed to the other part of the specimen as may be seen in Figure 5. Reinforcement strains were measured using electronical foil strain gauges glued to the reinforcement at the shear plane.

Figure 4: Test setup

Figure 5: Test instrumentations

## 4. EXPERIMENTAL RESULTS AND DISCUSSION

A comparison between the test specimens with respect to their experimental ultimate loads and their axial stiffness is shown in Figure 6 and 7. If the control specimen SS1 was taken as a reference with its ultimate load and stiffness taken as 1, specimens FS2, FA2 and FH3 would have ultimate strengths of 1.23, 0.76 and 0.99 of that of specimen SS1 respectively, and axial stiffness's of 0.63, 0.22 and 0.57 of the axial stiffness of SS1, respectively. These observations indicate the exceptional performance of the GFRP reinforcement in this particular application where the stiffness appears not to have a major role in the ultimate shear transfer strength within the range of these specimens. Despite the low tensile modulus of elasticity of the GFRP reinforcement, which usually controls the design of FRP reinforced concrete structures, the results were inspiring as compared to the control specimen. The shear load capacities and the reinforcement stiffness's of the test specimens are compared with respect to the control specimen in Figures 6 and 7, respectively. It can be seen from these figures that although the GFRP reinforced specimens had a reinforcement stiffness much lower than the stiffness of the steel reinforcement in the control specimen, the shear load resistance of these specimens was not proportional to the stiffness and was compatible to the control specimen. For example, specimen FS2 (i.e. the specimen with two GFRP stirrups) has a reinforcement stiffness (EA) equals to 0.63 of the stiffness of the one steel stirrup provided in the control specimen, SS1. However, FS2 had a strength about 23% higher than that of SS1.

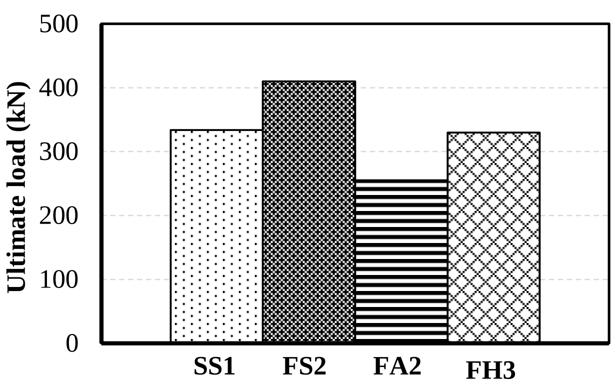


Figure 6: Ultimate load of test specimens

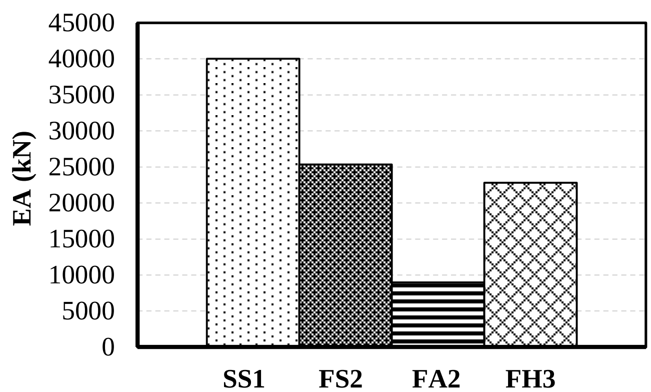


Figure 7: Axial stiffness of shear reinforcement

Figure 8 illustrate the shear load-slip behaviour of the specimens. It is noted that post failure ductility is the main behaviour that distinct the FRP specimens from the control specimen SS1. Relative slips were 0.45, 0.15, 0.14 and 0.14 for FS2, FA2, FH3 and SS1, respectively. The load-slip diagrams show the change in the behaviour when different shapes of GFRP reinforcement are provided with almost the same area as discussed in the following. The control specimen SS1 was also included as reference of these diagrams. The failure of the specimen SS1 was relatively brittle where the load decreased rapidly with the increasing slip right after the cracking of the shear plane. The reinforcement ratio of this specimen equals to 0.17% which is very close to the minimum shear transfer reinforcement of 0.15% suggested by Mattock and Kaar (1961). For this ratio, there was no additional strength provided after the cracking. In other words, the shear friction theory was not utilized, in this case, due to the lack of the compressive stresses that are needed to mobilize the friction at the interface. Both of FA2 and FH3 failed in a similar manner, so the failure occurred at the cracking of the interface, but they exhibited higher post failure ductility, as maybe observed from Figure 8. Hanson (1960) recommended a slip limit equal to 0.13 mm at which the interface failure maybe considered to have occurred. This finding is consistent with slips at the ultimate load of FA2, and FH3 which had reinforcement ratios of 2.2 and 3.3% respectively. However, it can be seen for the load-slip curve associated with specimen FS2 that a post cracking resistance was developed and the slip at failure was higher (0.45 mm). After cracking occurred at about 73% of the ultimate load, an additional 112 kN of shear resistance was delivered by the mean of friction with the increasing of the relative slip between the blocks of the specimen.

Specimens FS2 and FH3 reported an exceptional ductility as compared to SS1 and FA2, where about 76 and 91% of the ultimate load was maintained up to the total failure of specimens FS2 and FH3 respectively. A shear resistance in both of the referred two specimens were developed at roughly 1-1.5 mm. This resistance is believed to be caused by the dowel action of the reinforcing bars across the interface. It worth to note that the dowel resistance of the steel reinforcement in SS1 was not developed until a slip of 3 mm was reached, as maybe seen in Figure 8. This finding is consistent with the observation made by Mattock and Hawkins (1969) in their investigation of the contribution of the dowel action in the shear transfer resistance.

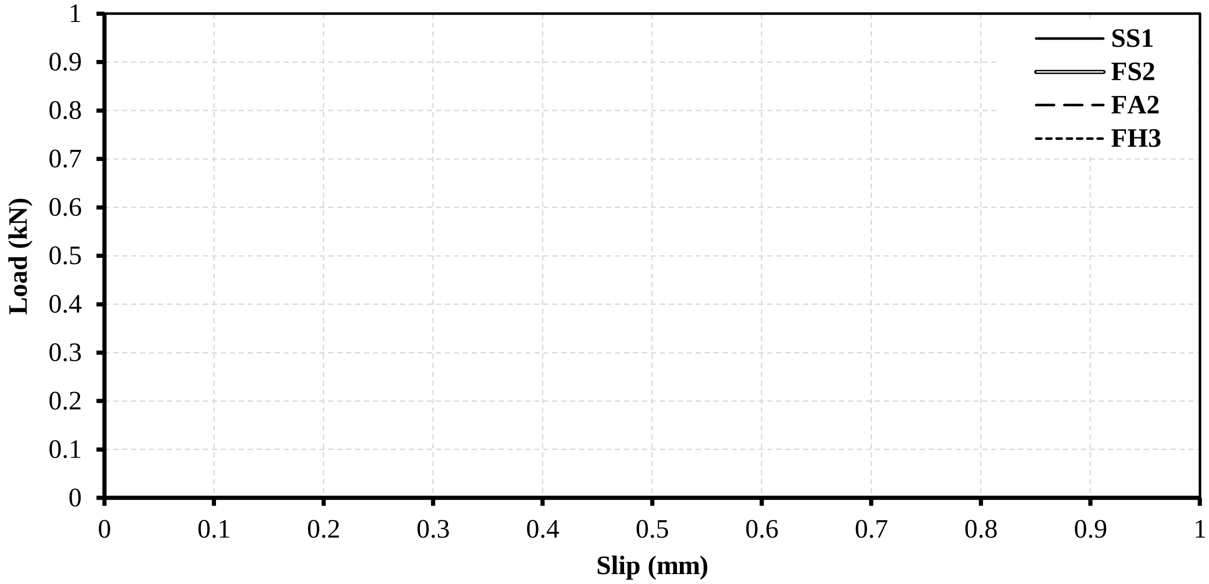

Figure 8: Load-slip behaviour of test specimens

Cracking of the shear plane of the steel specimen SS1 was sudden and associated with an extensive spalling of the concrete at the shear interface as shown in Figure 9(a). Whereas, hairy crack developed along the interface of the FRP reinforced specimens and developed slowly with increasing load up to failure. However, quickly after the crack was initiated, the concrete cover of one of the GFRP angles spalled in specimen FA2. This spalling appeared as a triangular crack with the shape of the reinforcement angle on one side of the specimen as can be seen in Figure 9(c). This side was later verified to be the one with less concrete cover after the specimen was cut open at its interface.

(a) SS1 (b) FS2 (c) FH3 (d) FA2

Figure 9: Failure mode of test specimens

## 5. CONCLUSION

The objective of this research was to explore the effectiveness of the use of GFRP reinforcement as shear transfer reinforcement across construction joints with concrete cast at different times, for better stimulation of the interfaces between precast girders and cast-in-place concrete. Different reinforcement ratios and shapes of the GFRP reinforcement were used. The interface surface of the joint was left as-cast with some aggregates protruded from the surface. A comprehensive analysis was conducted and the following conclusions were drawn:

- Glass Fiber Reinforced Polymers showed an outstanding shear resistance, even with much lower axil stiffness as compared to the stiffness of the steel reinforcement.
- Post failure ductility is the main characteristic of the FRP reinforced specimens that distinguish them from the control specimen with steel reinforcement.
- When a small GFRP reinforcement ratio ($\rho_v < 0.44$ %) was provided, the failure occurred immediately after the cracking of the interface.
- Dowel action resistance associated with shearing of the GFRP bars at the interface level was observed to take place at a relative slip in the range of 1 to 1.5 mm. being associated with these high values of slip, the dowel action resistance may not be suitable to be incorporated in the strength design of the concrete interface.
- For properly reinforced concrete-to-concrete interfaces with GFRP reinforcement with a ratio equals to or greater than 0.44%, the shear friction theory appeared to be applicable where an additional resistance can be delivered after cracking by the mean of friction.

## ACKNOWLEDGEMENT

The authors wish to express their gratitude and sincere appreciation for the contribution made by Pultral for supplying the experimental materials. The help received from the technical staff of the Structural Laboratory in the Department of Civil and Environmental Engineering at the University of Windsor is also acknowledged.

## REFRENCEES

ACI Committee 711. (1953). Minimum Standard Requirments for Precast Concrete Floor Units (ACI 711-53). *Journal of the American Concrete Institute* , 7-8.

Anderson, A. R. (1960). Composite Deisgns in Precast and Cast-in-Place Concrete. *Progressive Architecture*, 172-179.

ASTM A615. (2015). *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement.* West Conshohocken, PA: ASTM International.

ASTM C39. (2015). *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.* West Conshohocken, PA: ASTM International.

ASTM C496. (2011). *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.* West Conshohocken, PA: ASTM International.

Birkeland, H. (1968). Class notes for coarse "Precast and Pestressed Concrete". *University of Britich Columbia*. (Cross referenced from Loov and Patnaik, 1994)

Birkeland, P., & Birkeland, H. (1966). Connections in Precast Concrete Construction. *ACI Journal*, 345-367.

Candaian Standard Association (CSA). (2004). *Design of Concrete Structures for Buildings CAN/CSA-A23.3-04.* Rexdale, Ontario: Canadian Standard Association.

Hanson, N. (1960). Precast-Prestressed Concrete Bridges 2.Horizontal Shear Connections. *PCA Journal, 2*(2), 38-58.

Hofbeck, J., Ibrahim, I., & Mattock, A. (1969). Shear Transfer in Reinforced Concrete. *ACI Journal*, *66*(2), 119-128.

Loov, R. (1978). Design of Precast Connections. *Paper presented at a seminar orgnized by Compa International Pte, Ltd*, 8 pages.

Loov, R. (1978, Sep 25-27). Design of Precast Connections. *Paper presented at seminar organized by Compa International Pte, Ltd*, 8 pages.

Loov, R., & Patnaik, A. (1994). Horizontal Shear Strength of Composite Concrete Beams With Rough Interface. *PCI Journal*, 48-69.

Loov, R., & Patnaik, A. (1994). Horizontal Shear Strength of Composite Concrete Beams with Rough Interfaces. *PCI Journal*, 48-69.

Mast, R. (1968). Auxiliary Reinforcement in Concrete Connections. *ASCE Journal*, 1485-1505.

Mattock, A., & Hawkins, N. (1972). Shear Transfer in Reinforced Concrete-Recent Research. *PCI Journal*, *17*(2), 55-75.

Mattock, A., & Kaar, P. (1961). Precast-Prestressed Concrete Bridges, 4. Shear Tests of Continous Girders. *PCI Journal*, 19-46.

Mattock, A., Johal, L., & Chow, H. (1975). Shear Transfer in Reinforced Concrete with Moment or Tension Across the Shear Plane. *PCI Journal*, 76-93.

Paulay, T., Park, R., & Phillips, M. (1974). Horizontal Construction Joints in Cast-in-Place Reinforced Concrete. *ACI Special Publication SP-4: Shear in Reinforced Concrete, Vol. 2, American Concrete Institute, Detroit* , 559-611.

Pianca, F., Schell, H., & Cautillo, G. (2005). The performance of epoxy coated reinforcement: experience of the Ontario ministry of transportation. *International Journal of Materials and Product Technology*, 286-308.

Randi, N. (1997). *Investigation on Transfer of Forces Between Old and New Concrete at Different Joint Roughness, PhD thesis* . Austria: University of Innsbruck.

V-Rod. (2016). *V-Rod Fiber Glass Reinforcement*. Retrieved from http://www.vrod.ca/en/Fiberglass-reinforcement/Civil-Engineering

Walraven, J., Frenay, J., & Pruijssers, A. (1987). Influence of Concrete Strength and Load Histrory on the Shear Friction Capacity of Concrete Members. *PCI Journal*, 66-84.